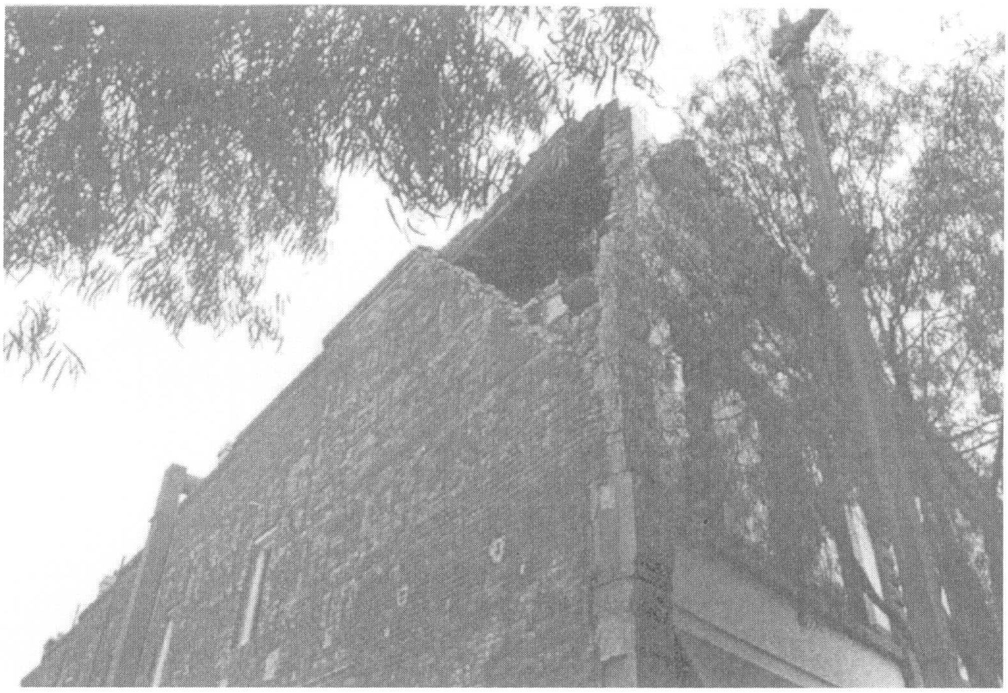


Figure 7-12 Photo of Corner Damage (from Rutherford and Chekene, 1990)

and the height-to-thickness ratio. The graph indicates that walls meeting the requirements have a "98% probability of survival." The authors did not provide relationships between damage and spectral acceleration or peak ground acceleration because input velocities were found to be a better predictor of wall performance. Therefore, it is difficult to relate damage to the spectral accelerations and nonlinear static procedures used in documents such as FEMA 273 for in-plane motions. As a result, the prescriptive *h/t* ratio concept is retained in this document.

7.2.17 Other Modes

A review of the literature provides a substantial number of specimens that cannot be easily placed in the above set of categories. Most common are tests that report "diagonal cracking" but do not specify stair-stepping bed-joint sliding, diagonal tension cracking, or diagonally-oriented compressive splitting cracks.

In addition, given the geometric complexity and material variation inherent in unreinforced masonry walls, localized stress concentrations can develop that are difficult to predict.

7.3 Unreinforced Masonry Evaluation procedures

7.3.1 Overview

This section contains evaluation procedures for analytical determination of expected behavior mode strengths and capacities. It is to be used in concert with the component guides contained in Section 7.5. Only in-plane and out-of-plane behavior of walls are addressed; for information on non-URM wall element behavior modes, see Section 7.2.

The analytical procedures described below help establish or confirm the expected inelastic mechanism of response so that component types and behavior modes are correctly identified. See previous portions of the document for details on how these capacity curves are to be used. The sophistication of the force-deformation relationship of a multi-story URM wall can vary widely, primarily depending on whether spandrel effects and global overturning effects are taken into consideration. Existing standards--such as ABK (1984), Division 88 (City of Los Angeles, 1985), RGA (SEAOSC, 1986), UCBC (ICBO, 1994) and FEMA 178

(BSSC, 1992)--do not have specific provisions for modeling of spandrels, and, in provisions for weak pier systems, these standards also do not provide provisions for the impact of global overturning on individual piers. FEMA 273 (ATC, 1997a) also does not provide explicit guidance for these issues.

For this document, it is considered acceptable to ignore potential spandrel and global overturning effects in a perforated wall element if spandrel damage is not observed in the field. In such a case, the spandrels are assumed to have sufficient capacity and the inelastic mechanism of response is assumed to be a weak pier system. Consequently, the nonlinear static analysis need only consider the force-displacement relationships of the piers in the wall element. See Section 7.3.2 for specific evaluation procedures.

See Section 7.3.3 for specific evaluation procedures for solid wall components.

If spandrel damage is observed, then the model of the wall should include spandrel components. In many cases, inelastic behavior in spandrels will transform an initial system of strong piers and weak spandrels into a system of weak piers and strong spandrels, as the strength of the spandrel diminishes. See Section 7.3.4 for an example and specific evaluation procedures.

In this document, out-of-plane wall and pier behavior are separated from in-plane behavior. Out-of-plane capacity and its potential reduction due to observed damage is to be evaluated and reported separately. See Section 7.3.5 for specific evaluation procedures. When significant out-of-plane damage is observed, it may have an effect on the wall for the force-deformation curve oriented parallel or in-plane to the wall. For such cases, the component guides in Section 7.5 contain λ-factors to apply for in-plane loading.

The analytical procedures require the determination of material properties which can be obtained through testing or by assumption and verification. FEMA 273 (ATC, 1997a) provides the scope and details of testing for determining v_{me} (in-place push tests), f'_{me} (extracted or mockup prism tests or in-situ flatjack tests), and modulus of elasticity E (extracted prisms or flatjack testing). Conservative default values are also given to be used in lieu of testing. To determine the value of average flat-wise compressive strength of the brick, use ASTM C-67.

7.3.2 Evaluation Procedures for In-Plane Behavior of Piers in Walls with Weak Pier - Strong Spandrel Mechanisms

Evaluation of pier capacity is a three-step process:

a. Step 1: Calculate Capacities for Individual Behavior Modes

Determine capacities for each of the following five values:

- Rocking (V_r):

$$V_r = 0.9\alpha P_{CE}(L/h_{eff}) \tag{7-3}$$

where:

α = factor equal to 0.5 for fixed-free cantilever wall, or equal to 1.0 for a fixed-fixed pier
P_{CE} = expected vertical axial compressive force per load combinations in FEMA 273 (ATC, 1997a)
L = length of the wall
h_{eff} = height to resultant of lateral force. For piers with regular opening, h_{eff} is the clear height of pier; for irregular openings, see Kingsley (1995). The parameter h_{eff} may be varied to reflect observed crack patterns. See Figure 7-14 for an example.

- Bed joint sliding with bond plus friction (V_{bjs1}) and with friction only (V_{bjs2}):

$$V_{bjs1} = v_{me}A_n \tag{7-4}$$

where:

v_{me} = bond plus friction strength of mortar, as defined in FEMA 273 (1997a)
A_n = area of net mortared/grouted section

and:

$$V_{bjs2} = v_{friction}A_n = [0.75(P_{CE}/A_n)/1.5][A_n] = 0.5P_{CE} \tag{7-5}$$

- Diagonal tension (V_{dt}):

$$V_{dt} = f'_{dt} A_n (\beta)(1 + f_{ae}/f'_{dt})^{1/2} \qquad (7\text{-}6)$$

where:

f'_{dt} = diagonal tension strength, assumed as v_{me}, per FEMA 273 (1997a)
β = 0.67 for L/h_{eff} <0.67, L/h_{eff} when 0.67 ≥ L/h_{eff} ≤ 1.0, and 1.0 when L/h_{eff} >1

- Toe crushing (V_{tc}):

$$V_{tc} = \alpha P_{CE}(L/h_{eff})(1 - f_{ae}/0.7f'_{me}) \qquad (7\text{-}7)$$

where:

f_{ae} = expected vertical axial compressive stress as defined in FEMA 273 (ATC, 1997a)
f'_{me} = expected masonry compressive strength

b. Step 2: Determine Predicted Behavior Mode and Capacity:

Differentiate piers by aspect ratio and applied vertical stress to determine which behavior mode is predicted as follows. Unless otherwise noted, force-displacement relationships are per FEMA 273 (ATC, 1997a).

Piers with aspect ratios of L/h_{eff} >1.25

- If V_{bjs1} is the lowest value and less than 0.75 of V_{tc} or V_r, then URM2B (bed joint sliding) is the predicted mode, with an initial capacity of V_{bjs1}.
- If V_{tc} or V_r are the lowest values and are less than 0.75 of V_{bjs1}, then URM1H (flexural cracking/toe crushing) is the predicted mode, and V_{tc} is the predicted capacity. Assume this mode is force-controlled.
- If V_{tc}, V_r, and V_{bjs1} are lower than V_{dt}, 0.75 V_{bjs1} ≤ V_{tc} ≤ V_{bjs1} and 0.75 V_{bjs1} ≤ V_r ≤ V_{bjs1}, then a sequence of URM1F (flexural cracking/toe crushing/bed joint sliding) is the predicted mode. Use V_{tc} for the initial capacity up to a "d" drift of 0.4% and V_{bjs2} for the final capacity from "d" to an "e" of 0.8%.
- If one of the categories above is not met, then the predicted capacity is the lowest of V_r, V_{bjs1}, V_{dt}, and V_{tc} and the associated behavior mode is as follows:
  - V_r: Mode URM2A (wall-pier rocking)
  - V_{bjs1}: Mode URM2B (bed joint sliding)
  - V_{dt}: Mode URM2K (preemptive diagonal tension)
  - V_{tc}: Mode URM2L (preemptive toe crushing)

Piers with aspect ratios of L/h_{eff} ≤ 1.25

- If V_r or V_{tc} are the lowest values and f_{ae} < 100 psi, then URM2A is the predicted mode with V_r as the initial capacity.
- If V_r or V_{tc} are the lowest values and f_{ae} ≥ 150 psi, then diagonal cracking with limited ductility, such as URM2G (flexural cracking/diagonal tension) is the predicted mode with V_{tc} as the capacity.
- If one of the categories above is not met, then the predicted capacity is the lowest of V_r, V_{bjs1}, V_{dt} and V_{tc} and the associated behavior mode is as follows:
  - V_r: Mode URM2A (wall-pier rocking)
  - V_{bjs1}: Mode URM2B (bed joint sliding)
  - V_{dt}: Mode URM2K (preemptive diagonal tension)
  - V_{tc}: Mode URM2L (preemptive toe crushing)

c. Step 3: Compare Predicted Mode with Observed Field Damage:

If field damage is consistent with predicted damage shown in the damage guide, then assume the component and damage classification and the capacity are correct.

If field damage is inconsistent with predicted damage shown in the damage guide, return to analysis at Step 1 and vary assumptions as to material properties and possible alternative modes. Consider, for example, for f'_{dt}, using 1/30 of the value of average flat-wise compressive strength of the brick in lieu of v_{me}. This test is standardized in ASTM C-67.

7.3.3 Evaluation Procedures for In-Plane Behavior of Solid Wall Components

Evaluation procedures for solid walls are similar to those for piers in walls with weaker pier-stronger spandrel mechanisms. Equations in Section 7.3.2 may be used, with the appropriate use of α=0.5. For the rocking equation in Section 7.3.2, the weight of the pier is ignored for simplicity, since it is assumed to be only a small fraction of the superimposed vertical load. When the weight of the wall represents a significant fraction of the vertical load, then the rocking equation may be modified as follows.

$$V_r = 0.9\alpha(P_{CE} + W_W)L/h_{eff} \qquad (7\text{-}8)$$

where:

α = factor equal to 0.5 for fixed-free cantilever wall
P_{CE} = expected vertical axial compressive force per load combinations in FEMA 273 (ATC, 1997a). This superimposed load is assumed to act at the center of the wall coincident with the location of the weight of the wall.
W_W = expected weight of the wall
L = length of the wall
h_{eff} = height to resultant of lateral force

7.3.4 Evaluation Procedures for In-Plane Behavior of Perforated Walls with Spandrel Damage

There is no methodology to analyze spandrels in the literature. As a placeholder until research is carried out, the following procedures have been developed. Procedures are given for estimating the moment and shear capacity of an uncracked spandrel, and for damaged walls that have experienced spandrel joint sliding or spandrel unit cracking. Examples are given of how to address the implications of spandrel cracking on in-plane behavior of perforated walls.

a. Capacities of an Uncracked Spandrel

Moment Capacity. The moment capacity of the uncracked spandrel is assumed to be derived from the interlock between the bed joints and collar joint at the interface between the pier and the spandrel. See Figure 7-13. An elastic stress distribution is assumed across the end of the spandrel with the neutral axis located at the centerline of the spandrel height. It is assumed that the bed joint and collar joint capacities can be linearly superimposed to produce a resultant force. Both tension and compressive resultants are assumed to be derived from the mortar shear strength. (Note that alternative formulations are possible that use the compressive strength of the masonry to develop the compressive force.) Irregularities due to header courses are ignored. The uncracked moment capacity M_{spun} is then the product of the resultant force and the effective distance between the resultant.

- Uncracked bed joint shear stress, (v_{bjun}):

$$v_{bjun} = 0.75(0.75\, v_{te} + \gamma P_{CE}/A_n)/1.5 \qquad (7\text{-}9)$$

where:

v_{te} = the average test value from in-place testing
P_{CE} = the expected vertical axial compressive force per load combinations in FEMA 273 (ATC, 1997a) at the adjacent pier
A_n = the area of net mortared/grouted section of the adjacent pier
γ = 0.5. This arbitrary value indicates that the vertical axial stress on the spandrel bed joints at the end of the spandrel is assumed to be approximately half of the axial stress within the pier above the pier/spandrel joint.

- Uncracked collar joint shear stress, (v_{cun}):

$$v_{cun} = 0.75(0.75\, v_{te} + \gamma P_{CE}/A_n)/1.5$$
$$= 0.375 v_{te} \qquad (7\text{-}10)$$

where:

v_{te} = the average test value from in-place testing

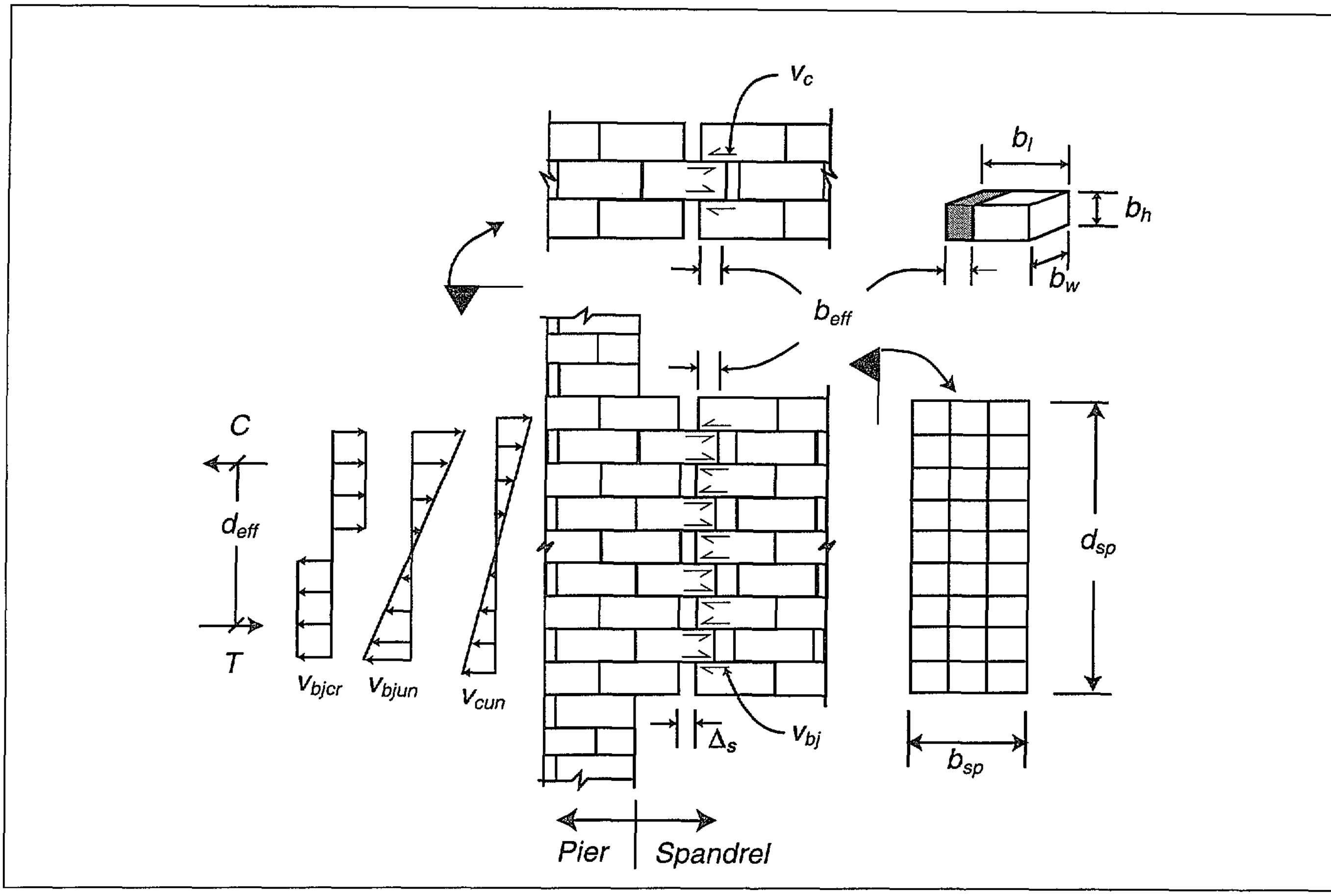


Figure 7-13 Spandrel Joint Sliding

P_{CE} = the expected vertical axial compressive force per load combinations in FEMA 273 (ATC, 1997a) at the adjacent pier

A_n = the area of net mortared/grouted section of the adjacent pier

γ = 0. This arbitrary value indicates that the axial stress on the spandrel collar joints at the end of the spandrel is assumed to be negligible.

- Effective length of interface for an uncracked spandrel, (b_{effun}):

$$b_{effun} = b_l/2 \tag{7-11}$$

where:

$b_l/2$ = half the length of the masonry unit

- Number of rows of bed joints, (NR):

$$NR = 0.5(d_{sp}/b_h) \tag{7-12}$$

Round NR down to the nearest whole number

where:

b_h = height of the brick unit plus the bed joint thickness

d_{sp} = depth of the spandrel

- Resultant tensile and compressive result forces, ($T=C$):

$$T = [(v_{bjun})(b_w)(b_{effun}) + (v_{cun})(b_h)(b_{effun})(NB\text{-}1)](\eta) \tag{7-13}$$

where:

b_w = width of the brick unit

b_h = height of the brick unit
NB = number of brick wythes
η = NR/2 or for more sophistication:
= $\Sigma_{i=1,NR}\ [(d_{sp}/2 - b_h(i)) / (d_{sp}/2 - b_h)]$

- Uncracked moment of spandrel, (M_{spun}):

$$M_{spun} = (d_{effun})\,(T) \qquad (7\text{-}14)$$

where:
d_{effun} = distance between T and C
= $(2/3)\,(d_{sp})$

Shear Capacity. The shear capacity of the spandrel is derived here from the equation for diagonal tensile capacity for the pier as follows.

- Diagonal tension (V_{spun}):

$$V_{spun} = f'_{dt}\, d_{sp}\, b_{sp}(\beta)\,(1 + f_{ae}/f'_{dt})^{1/2} \qquad (7\text{-}15)$$

where:
f'_{dt} = diagonal tension strength, assumed as v_{me}, per FEMA 273 (ATC, 1997a)
β = 0.67 for $L_{sp}/d_{sp} < 0.67$, L_{sp}/d_{sp} when $0.67 \geq L_{sp}/d_{sp} \leq 1.0$, and 1.0 when $L_{sp}/d_{sp} > 1$
L_{sp} = length of spandrel
f_{ae} = expected horizontal axial stress in the pier
= 0, unless known.
b_{sp} = width of spandrel

With f_{ae}=0, this equation then reduces to:

$$V_{spun} = f'_{dt}\, d_{sp}\, b_{sp}(\beta) \qquad (7\text{-}16)$$

b. Capacities of a Cracked Spandrel with Spandrel Joint Sliding

Moment Capacity. The moment capacity of the cracked spandrel with spandrel joint sliding is derived similar to the procedure given for an uncracked spandrel. Again see Figure 7-13.

- Cracked bed joint shear stress, (v_{bjcr}):

$$v_{bjcr} = 0.75(\varepsilon v_{te} + \gamma P_{CE}/A_n)/1.5$$
$$= 0.25\, P_{CE}/A_n \qquad (7\text{-}17)$$

where:
v_{te} = the average test value from in-place testing
ε = 0. The bond strength of the mortar is assumed to be lost.
P_{CE} = the expected vertical axial compressive force per load combinations in FEMA 273 (ATC, 1997a) at the adjacent pier
A_n = the area of net mortared/grouted section of the adjacent pier
γ = 0.5. This arbitrary value indicates that the vertical axial stress on the spandrel bed joints at the end of the spandrel is assumed to be approximately half of the axial stress within the pier above the pier/spandrel joint.

- Cracked collar joint shear stress, (v_{ccr}):

$$v_{ccr} = 0.75\,(\varepsilon\, v_{te} + \gamma P_{CE}/A_n)/1.5 = 0 \qquad (7\text{-}18)$$

where:
ε = 0. The bond strength of the mortar is assumed to be lost.
γ = 0. This arbitrary value indicates that the axial stress on the spandrel collar joints at the end of the spandrel is assumed to be negligible.

- Effective length of interface for a cracked spandrel, (b_{effcr}):

$$b_{effcr} = b_l/2 - \Delta_s \qquad (7\text{-}19)$$

where:
$b_l/2$ = half the length of the masonry unit
Δ_s = average slip (can be estimated as average opening width of open head joint)

- Number of rows of bed joints, (NR):

$$NR = 0.5(d_{sp}/b_h) \qquad (7\text{-}20)$$

Round NR down to the nearest whole number

where:
b_h = height of the brick unit plus the bed joint thickness
d_{sp} = depth of the spandrel

- Resultant tensile and compressive result forces, $(T{=}C)$:

$$T = (v_{bjcr})\,(b_w)\,(b_{effcr})\,(NR) \qquad (7\text{-}21)$$

where:
b_w = width of the brick unit

- Cracked moment of spandrel, (M_{spcr}):

$$M_{spcr} = (d_{effcr})\,(T) \qquad (7\text{-}22)$$

where:
d_{effcr} = distance between T and C
= $(1/2)(d_{sp})$

Shear Capacity. The shear capacity of a cracked spandrel with spandrel joint sliding is assumed to be the same as that of an uncracked spandrel provided keying action between bricks remains present at the end of the spandrel. Shear is resisted by bearing on the bed joints of the interlocked units.

c. Capacities of a Cracked Spandrel with Spandrel Unit Cracking

The moment and shear capacity of an cracked spandrel with spandrel unit cracking is derived similar to the procedure given above for an uncracked spandrel. The only modification is that the effective depth of the spandrel is reduced to only the amount of uncracked masonry remaining.

d. Examples of the Implications of Spandrel Cracking

Figure 7-14 shows a wall line with some cracking at the ends of spandrels. This section qualitatively discusses corner damage and gives quantitative procedures for assessing the impact of spandrel cracking on adjacent piers.

Corner Damage. One of the potential causes of corner damage is shown at the top of Figure 7-14 where the moment and shear at the end of the spandrel are resisted only by the weight of the masonry near the joint, direct tension on the head joints and bed joints, and shear in the collar joints. When these fairly weak capacities are exceeded, a diagonally-oriented crack propagates from the upper corner of the opening across the last pier. Since the crack is inclined, the effective height of the last pier is increased. For loading to the right, it may be appropriate to move the superimposed dead load closer to the center of the pier in the evaluation of the pier rocking capacity of the last pier.

Multistory Pier Rocking. As noted above, it is assumed in this document that if there is no spandrel damage, then a weak pier-strong spandrel model should be used. On the other hand, if the spandrels are fully cracked, then there will be no bending rigidity provided by the spandrel, and the pier rocking should be assessed using a multistory pier. When the spandrels have a reduced capacity, it is necessary to determine the capacity of both the typical single story pier rocking and the multistory case. Figure 7-14 shows an example. To assess the multistory rocking capacity of Mechanism 1 in the figure the following procedure may be used:

- Assume that the shear imparted by the spandrel on the pier (V_{sp}) is

$$V_{sp} = 2M_{sp} / L_{sp} \qquad (7\text{-}23)$$

where:
M_{sp} = the bending capacity of the spandrel as determined from previous sections
L_{sp} = the length of the spandrel

- Assume a distribution of acceleration within the wall line. Using ABK (1984) assumptions, the acceleration is uniform up the wall. Further assume for this example that loads tributary to each level are the same so that $V_{rR} = V_{r2}$

- Sum the moments around the pier toe at the first story so that:

$$\begin{aligned}\Sigma M &= 0\\ &= (V_{rR})(h_1 + h_2 + d_{sp}) + (V_{r2})\,(h_1 + d_{sp}/2) - \\ &\quad (P_{DLR})\,(0.9L) - (V_{sp} + (1/2)\,(P_{DL2}))(0.9L)\\ &\quad - 2M_{sp} \qquad (7\text{-}24)\end{aligned}$$

Substituting for V_{r2} and V_{sp} gives:

$$V_{rR} = [0.9\,(P_{DLR} + P_{DL2}/2 + 2M_{sp}/L_{sp}) + 2M_{sp}] \,/\, [2h_1 + h_1 + 3d_{sp}/2] \qquad (7\text{-}25)$$

where:
V_{rR} = the shear at the second story
h_1 = height of the first story pier
h_2 = height of the second story pier
d_{sp} = depth of the spandrel

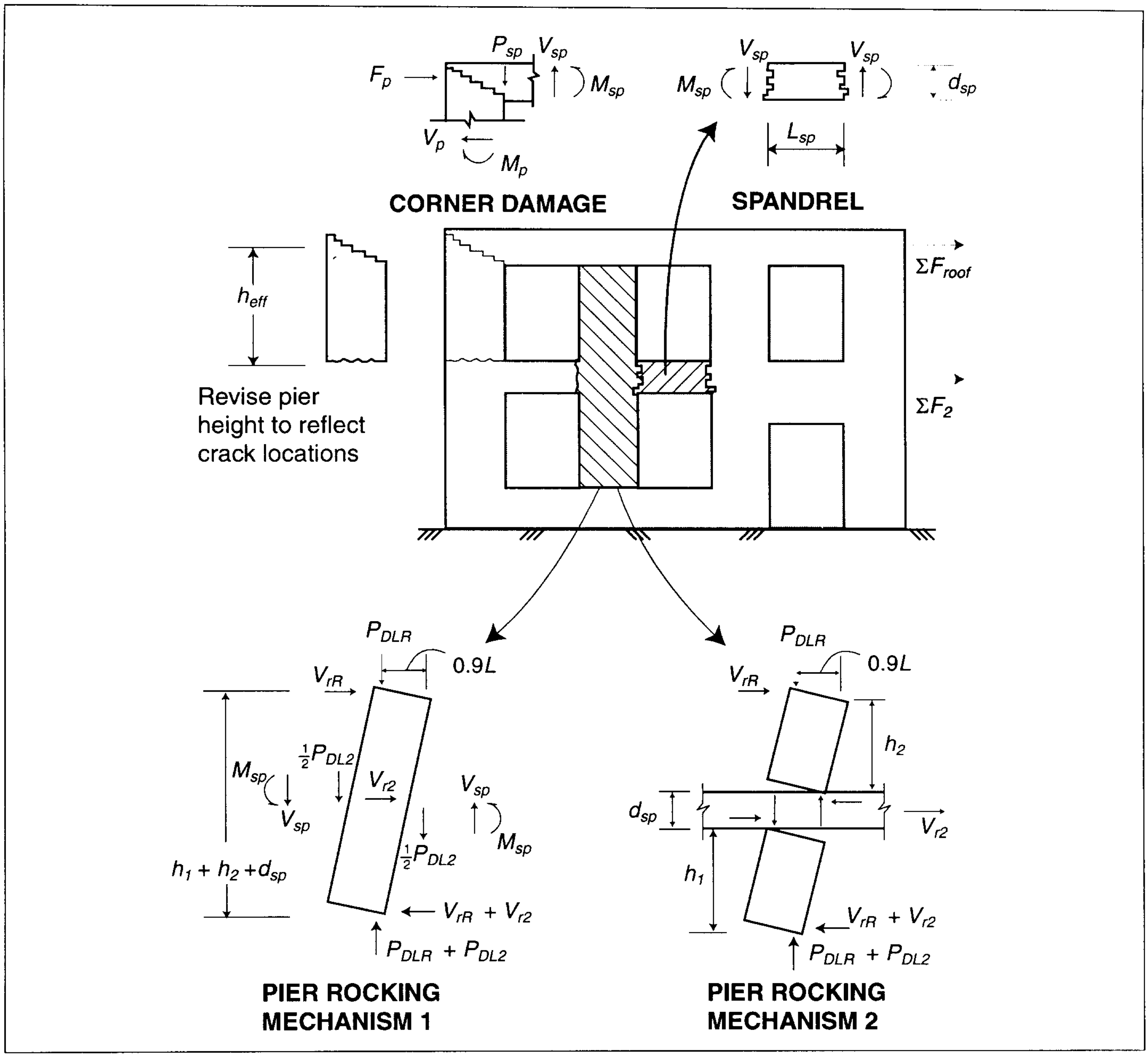


Figure 7-14 Implications of Spandrel Cracking

P_{DLR}= the expected gravity load at the roof

P_{DL2}= the expected gravity load at the second story, assumed here to be split equally on both sides of pier. Note that the pier weight is ignored for simplicity.

Single Story Pier Rocking. The rocking capacity of the piers in Mechanism 2 is given here. For the second story:

$$V_{rR}=0.9\mathrm{L}\,(P_{DLR})\,/\,h_2 \qquad (7\text{-}26)$$

For the first story:

$$(V_{rR} + V_{r2})(h_1) = (P_{DLR} + P_{DL2})\,(0.9L) \qquad (7\text{-}27)$$

Substituting for V_{r2} gives:

$$V_{rR}=0.9L\,[(P_{DLR} + P_{DL2})\,/\,2h_1 \qquad (7\text{-}28)$$

Governing Mode. To determine the governing mode of behavior, compare the values for V_{rR} for multi-story and single-story rocking. The lowest rocking capacity will govern for determining the pier rocking strength.

Shear capacities for bed joint sliding, toe crushing and diagonal tension should then be compared. Use the equations in Section 7.3.2.

7.3.5 Evaluation Procedures for Out-of-Plane Behavior of Wall and Pier Components

Prescriptive strength and deformation acceptance criteria for out-of-plane wall demands are contained in FEMA 273 (ATC, 1997a). For the immediate occupancy performance level, flexural stresses should not exceed the tensile capacity of the wall. Thus, for any level of damage above Insignificant, where by definition some flexural cracking has occurred, the damaged wall should be considered as not meeting the immediate occupancy performance level.

For the collapse prevention and life safety performance levels, Table 7-4 of FEMA 273 tabulates permissible h/t ratios for walls without prior damage. For damaged walls, the Component Guides in this volume specify $\lambda_{h/t}$-factors which, when multiplied by the ratios in Table 7-4 (FEMA 273), give permissible h/t ratios for damaged walls.

The Component Guide also gives λ-factors for use when the damaged wall is a component in a force deformation curve oriented parallel to the length of the wall.

7.4 Symbols for Unreinforced Masonry

Symbols used in the unreinforced masonry sections of FEMA 306 and 307 are the same as those given in Section 7.9 of FEMA 273, except for the following additions and modifications.

C	Resultant compressive force in a spandrel, lb
L_{sp}	Length of spandrel, in.
M_{spcr}	Expected moment capacity of a cracked spandrel, lb-in.
M_{spun}	Expected moment capacity of an uncracked spandrel, lb-in.
V_{spcr}	Expected diagonal tension capacity of a cracked spandrel, lb
V_{spun}	Expected diagonal tension capacity of an uncracked spandrel, lb
NB	Number of brick wythes in a spandrel
NR	Number of rows of bed joints in a spandrel
T	Resultant tensile force in a spandrel, lb
V_{bjs1}	Expected shear strength of wall or pier based on bed joint shear stress, including both the bond and friction components, lb
V_{bjs2}	Expected shear strength of wall or pier based on bed joint shear stress, including only the friction component, lb
V_{sp}	Shear imparted on the spandrel by the pier, lb
V_{dt}	Expected shear strength of wall or pier based on diagonal tension using v_{me} for f'_{dt}, lb
V_{tc}	Expected shear strength of wall or pier based on toe crushing using v_{me} for f'_{dt}, lb
W_w	Expected weight of a wall, lb
b_{effcr}	Effective length of interface for a cracked spandrel, in.
b_{effun}	Effective length of interface for an uncracked spandrel, in.
b_h	Height of masonry unit plus bed joint thickness, in.
b_l	Length of masonry unit, in.
b_w	Width of brick unit, in.
d_{sp}	Depth of spandrel, in.
d_{effcr}	Distance between resultant tensile and compressive forces in a cracked spandrel, in.
d_{effun}	Distance between resultant tensile and compressive forces in an uncracked spandrel, in.
f'_{dt}	Masonry diagonal tension strength, psi
v_{bjcr}	Cracked bed joint shear stress, psi
v_{bjun}	Uncracked bed joint shear stress in a spandrel, psi
v_{ccr}	Cracked collar joint shear stress in a spandrel, psi
v_{cun}	Uncracked collar joint shear stress in a spandrel, psi
β	$=0.67$ when $L/h_{eff}<0.67$, $=L/h_{eff}$ when $0.67 \le L/h_{eff} \le 1.0$, and $=1.0$ when $L/h_{eff}>1$
Δ_s	Average slip at cracked spandrel (can be estimated as average opening width of open head joint), in.
ε	Factor for estimating the bond strength of the mortar in spandrels
γ	Factor for coefficient of friction in bed joint sliding equation for spandrels
η	Factor to estimate average stress in uncracked spandrel. Equal to NR/2 or, for more sophistication, use $\Sigma_{i=1,NR}\ [(d_{sp}/2 - b_h(i))/(d_{sp}/2 - b_h)]$
$\lambda_{h/t}$	Factor used to estimate the loss of out-of-plane wall capacity to damaged URM walls

μ_Δ Displacement ductility demand for a component, used in Section 5.3.4, and discussed in Section 6.4.2.4 of FEMA 273. Equal to the component deformation corresponding to the global target displacement, divided by the effective yield displacement of the component (which is defined in Section 6.4.1.2B of FEMA 273).

7.5 Unreinforced Masonry Component Guides

The following Component Damage Classification Guides contain details of the behavior modes for unreinforced masonry components. Included are the distinguishing characteristics of the specific behavior mode, the description of damage at various levels of severity, and performance restoration measures. Information may not be included in the Component Damage Classification Guides for certain damage severity levels; in these instances, for the behavior mode under consideration, it is not possible to make refined distinctions with regard to severity of damage. See also Section 3.5 for general discussion of the use of the Component Guides and Section 4.4.3 for information on the modeling and acceptability criteria for components.

URM2A	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**URM**
		Component Type:	**Weaker Pier**
		Behavior Mode:	**Wall-Pier Rocking**

How to distinguish behavior mode:

By observation:

Rocking-critical piers form horizontal flexural cracks at the top and bottom of piers. Because the cracks typically close as the pier comes back to rest at the end of ground shaking, these cracks can be quite subtle when only a few cycles of rocking have occurred and when pier drift ratios during shaking were small. As damage increases, softening of the pier can occur due to cracking, and the pier may begin to "walk" out-of-plane at the top and bottom. At the highest damage levels, crushing of units at the corners can occur.

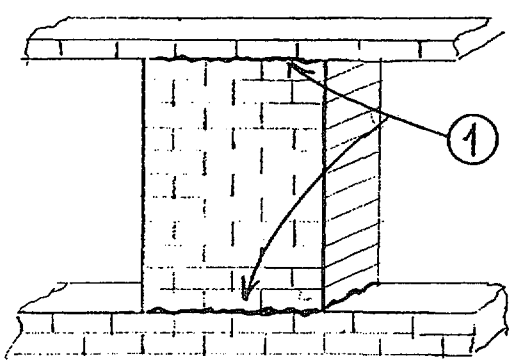
By analysis:

As damage increases to the Moderate level and beyond, some small cracking within the pier may occur. Confirm by analysis that rocking governs over diagonal tension and bed joint sliding.

Caution: If horizontal cracks are located directly below wall-diaphragm ties, damage may be due to bed joint sliding associated with tie damage. If a horizontal crack is observed at midheight of the pier, see URM1M.

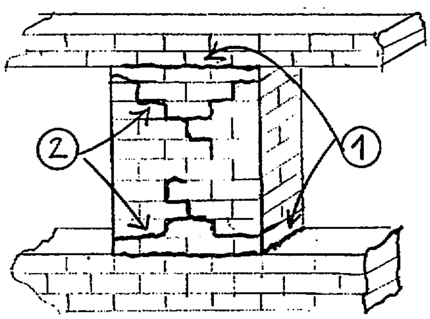
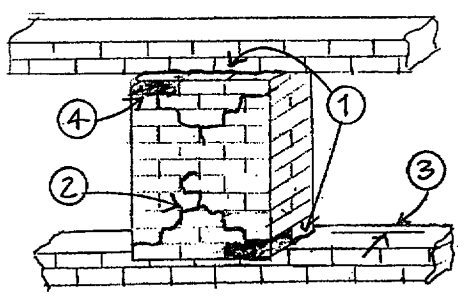
Refer to Evaluation Procedures for:

- In-plane wall behavior: See Section 7.3.2.

Level	Description of Damage	Typical Performance Restoration Measures
Insignificant $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$ $\mu_\Delta \leq 1.5$	*Criteria:* • Hairline cracks/spalled mortar in bed joints at top and bottom of pier. *Typical Appearance:* 	Not necessary for restoration of structural performance. (Measures may be necessary for restoration of nonstructural characteristics.)

COMPONENT DAMAGE
CLASSIFICATION GUIDE **continued**

URM2A

Level	**Description of Damage**	**Typical Performance Restoration Measures**
Slight	Not used.	
Moderate $\lambda_K = 0.6$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$ $\Delta/h_{eff} \leq$ h_{eff}/L_{eff}* 0.4%	*Criteria:* 1. Hairline cracks/spalled mortar in bed joints at top and bottom of pier. 2. Possible hairline cracking/spalled mortar in bed joints within piers. *Typical Appearance:* 	Replacement or enhancement is required for full restoration of seismic performance. For partial restoration of performance: • Repoint spalled mortar. λ_K* = 0.8 λ_Q* = 0.9 λ_D* = 1.0
Heavy $\lambda_K = 0.4$ $\lambda_Q = 0.8$ $\lambda_D = 0.7$ $\Delta/h_{eff} \leq$ h_{eff}/L_{eff}* 0.8%	*Criteria:* 1. Hairline cracks/spalled mortar in bed joints at top and bottom of pier, plus one or more of: 2. Hairline cracking/spalled mortar in bed joints within piers, but bed joints typically do not open. 3. Possible out-of-plane or in-plane movement at top and bottom of piers ("walking"). 4. Crushed/spalled bricks at corners of piers. *Typical Appearance:* 	Replacement or enhancement is required for full restoration of seismic performance. For partial restoration of performance: • Replace/drypack damaged units • Repoint spalled mortar • Inject cracks λ_K* = 0.8 λ_Q* = 0.9 λ_D* = 1.0
Extreme	*Criteria:* • Vertical load-carrying ability is threatened. *Typical Indications* • Significant out-of-plane or in-plane movement at top and bottom of piers ("walking"). • Significant crushing/spalling of bricks at corners of piers.	• Replacement or enhancement required.

URM2B	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	URM
		Component Type:	Weaker Pier
		Behavior Mode:	Bed Joint Sliding

How to distinguish behavior mode:

By observation:

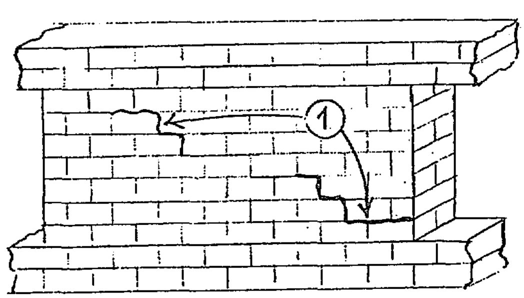
In this type of behavior, sliding occurs on bed joints. Commonly observed both in the field and in experimental tests, there are two basic forms: sliding on a horizontal plane, and a stair-stepped diagonal crack where the head joints open and close to allow for movement on the bed joint. Note that, for simplicity, the figures below only show a single crack, but under cyclic loading, multiple cracks stepping in each direction are possible. Pure bed joint sliding is a ductile mode with significant hysteretic energy absorption capability. If sliding continues without leading to a more brittle mode such as toe crushing, then gradual degradation of the cracking region occurs until instability is reached. Theoretically possible, but not widely reported, is the case of stair-stepped cracking when sliding goes so far that an upper brick slides off a lower unit.

By analysis:

Stair-stepped cracking may resemble a form of diagonal tension cracking; confirm by analysis that bed joint sliding governs over diagonal tension.

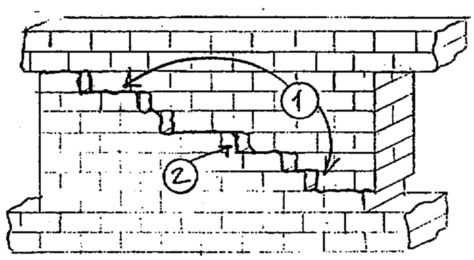
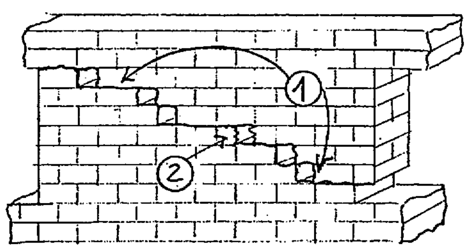
Refer to Evaluation Procedures for:

- In-plane wall behavior: See Section 7.3.2.

Level	Description of Damage	Typical Performance Restoration Measures
Insignificant $\lambda_K = 0.9$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$ $\mu_\Delta \leq 1.5$	*Criteria:* 1. Hairline cracks/spalled mortar in head and bed joints either on a horizontal plane or in a stair-stepped fashion have been initiated, but no offset along the crack has occurred and the crack plane or stair-stepping is not continuous across the pier. 2. No cracks in masonry units. *Typical Appearance:*  1	Not necessary for restoration of structural performance. (Measures may be necessary for restoration of nonstructural characteristics.)

COMPONENT DAMAGE
CLASSIFICATION GUIDE **continued**

URM2B

Level	**Description of Damage**	**Typical Performance Restoration Measures**
Slight	Not used.	
Moderate $\lambda_K = 0.8$ $\lambda_Q = 0.6$* $\lambda_D = 1.0$ *As an alternative, calculate as V_{bjs2}/V_{bjs1} $\Delta/h_{eff} \leq 0.4\%$	*Criteria:* 1. Horizontal cracks/spalled mortar at bed joints indicating that in-plane offset along the crack has occurred and/or opening of the head joints up to approximately 1/4", creating a stair-stepped crack pattern. 2. 5% of courses or fewer have cracks in masonry units. *Typical Appearance:* 	Replacement or enhancement is required for full restoration of seismic performance. For partial restoration of performance: • Repoint spalled mortar and open head joints. • Inject cracks and open head joints $\lambda_K* = 0.8$ $\lambda_Q* = 0.8$* $\lambda_D* = 1.0$* *In some cases, grout injection may actually increase strength, but decrease deformation capacity, by changing behavior from bed joint sliding to a less ductile behavior mode (see FEMA 307, Section 4.1.3).
Heavy $\lambda_K = 0.6$ $\lambda_Q = 0.6$* $\lambda_D = 0.9$ *As an alternative, calculate as V_{bjs2}/V_{bjs1} $\Delta/h_{eff} \leq 0.8\%$	*Criteria:* 1. Horizontal cracks/spalled mortar on bed joints indicating that in-plane offset along the crack has occurred and/or opening of the head joints up to approximately 1/2", creating a stair-stepped crack pattern. 2. 5% of courses or fewer have cracks in masonry units. *Typical Appearance:* 	Replacement or enhancement is required for full restoration of seismic performance. For partial restoration of performance: • Repoint spalled mortar and open head joints. • Inject cracks and open head joints. $\lambda_K* = 0.8$ $\lambda_Q* = 0.8$* $\lambda_D* = 1.0$* *In some cases, grout injection may actually increase strength, but decrease deformation capacity, by changing behavior from bed joint sliding to a less ductile behavior mode (see FEMA 307, Section 4.1.3).
Extreme	*Criteria:* • Vertical load-carrying ability is threatened. *Typical Indications* • Stair-stepped movement is so significant that upper bricks have slid off their supporting brick. • Cracks have propagated into a significant number of courses of units. • Residual set is so significant that portions of masonry at the edges of the pier have begun or are about to fall.	• Replacement or enhancement required.

URM3D	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**URM**
		Component Type:	**Weaker Spandrel**
		Behavior Mode:	**Spandrel Joint Sliding**

How to distinguish behavior mode:

By observation:

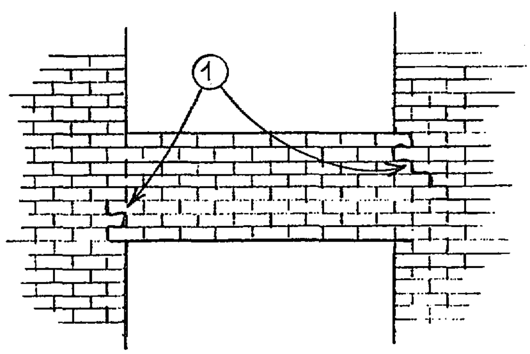
Commonly observed in the field in running bond masonry, this form of bed joint sliding is characterized by predominantly vertical cracks at the ends of the spandrel, which look like interlocked fingers being pulling apart. This mode can be relatively ductile and allow for significant drift, provided a reliable lintel is present. As the spandrel displaces, the nonlinear mechanism of response may move to other portions of the wall such as the piers.

By analysis:

No analysis is typically necessary to distinguish this mode. Analytical procedures are provided to estimate the reduction in capacity due to damage.

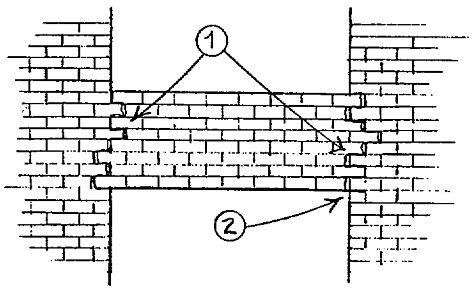
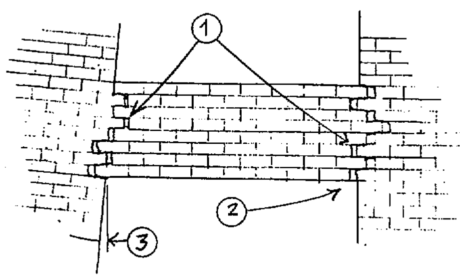
Refer to Evaluation Procedures for:

- In-plane wall behavior: See Section 7.3.4.

Level	Description of Damage	Typical Performance Restoration Measures
Insignificant $\lambda_K = 0.9$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$ $\mu_\Delta \leq 1.5$	*Criteria:* 1. Staggered hairline cracks/spalled mortar in head and bed joints in up to 3 courses at the ends of the spandrel. No cracks in units. *Typical Appearance:*  	Not necessary for restoration of structural performance. (Measures may be necessary for restoration of nonstructural characteristics.)
Slight	Not used	

COMPONENT DAMAGE
CLASSIFICATION GUIDE **continued**

URM3D

Level	**Description of Damage**	**Typical Performance Restoration Measures**
Moderate $\lambda_K = 0.8$ $\lambda_Q = 0.4^1$ $\lambda_D = 0.9$ 1. As an alternative, calculate per Section 7.3.4	*Criteria:* 1. Staggered hairline cracks/spalled mortar at the ends of the spandrel in head and bed joints indicating that in-plane offset along the crack has occurred and opening of the head joints up to approximately 1/4". No cracks in units. 2. No vertical slip of the spandrel. *Typical Appearance:* 	Replacement or enhancement is required for full restoration of seismic performance. For partial restoration of performance: • Repoint spalled mortar and open head joints. • Inject cracks and open head joints. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 0.8^1$ $\lambda_D{}^* = 1.0^1$ 1. In some cases, grout injection may actually increase strength, but decrease deformation capacity, by changing behavior from bed joint sliding to a less ductile behavior mode (see FEMA 307, Section 4.1.3).
Heavy $\lambda_K = 0.6$ $\lambda_Q = 0.4^1$ $\lambda_D = 0.9$ 1. As an alternative, calculate per Section 7.3.4	*Criteria:* 1. Staggered hairline cracks/spalled mortar at the ends of the spandrel in head and bed joints, indicating that in-plane offset along the crack has occurred and opening of the head joints up to approximately 1/2". No cracks in units. 2. Possibly some deterioration of units at bottom ends of spandrel, but no vertical slip of the spandrel. 3. Possibly spandrel rotation with respect to the pier. *Typical Appearance:* 	Replacement or enhancement is required for full restoration of seismic performance. For partial restoration of performance: • Repoint spalled mortar and open head joints • Inject cracks and open head joints $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 0.8^1$ $\lambda_D{}^* = 1.0^1$ 1. In some cases, grout injection may actually increase strength, but decrease deformation capacity, by changing behavior from bed joint sliding to a less ductile behavior mode (see FEMA 307, Section 4.1.3).
Extreme	*Criteria:* • Vertical load-carrying ability is threatened. *Typical Indications* • Sliding and/or deterioration of the units is so significant that keying action between bricks is lost. • Lintel support has separated from the pier.	• Replacement or enhancement required.

URM1F	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**URM**
		Component Type:	**Solid Wall**
		Behavior Mode:	**Flexural Cracking/ Toe Crushing/Bed Joint Sliding**

How to distinguish behavior mode:

By observation:

This type of moderately ductile behavior has been experimentally observed in walls with $L/h_{eff} \approx 1.7$ in which bed joint sliding and toe crushing strength capacities are similar. Damage occurs in the following sequence. First, flexural cracking occurs at the heel of the wall. Then diagonally-oriented cracks appear at the toe of the wall, typically accompanied by spalling and crushing of the units. In some cases, toe crushing is immediately followed by a steep inclined crack propagating upward from the toe. Next, sliding occurs along a horizontal bed joint near the base of the wall, accompanied in some cases by stair-stepped bed joint sliding at upper portions of the wall. With repeated cycles of loading, diagonal cracks increase. Eventually, crushing of the toes or excessive sliding leads to failure.

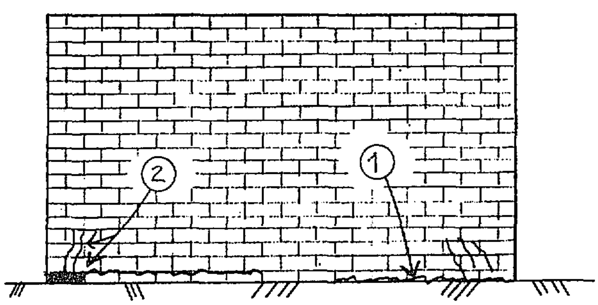
By analysis:

At higher damage levels, cracking may be similar to URM1H; however, in URM1F, the bed joint sliding will occur at the base of the wall, in addition to the center of the wall. Confirm by analysis that bed joint sliding capacities are sufficiently low to trigger URM1F.

Caution: At low damage levels, flexural cracking may be similar to cracking that occurs in other modes.

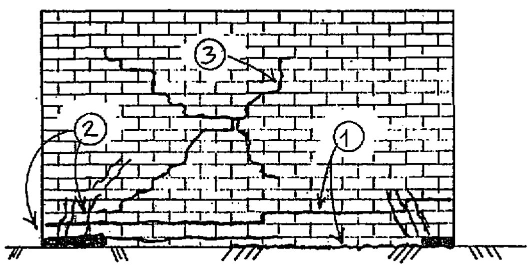
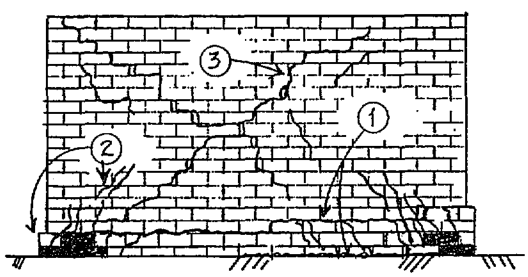
Refer to Evaluation Procedures for:

- In-plane wall behavior: See Section 7.3.2

Level	Description of Damage	Typical Performance Restoration Measures
Insignificant $\lambda_K = 1.0$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$ $\mu_\Delta \leq 1.5$	*Criteria:* 1. Horizontal hairline cracks in bed joints at the heel of the wall. 2. Possibly diagonally-oriented cracks and minor spalling at the toe of the wall. *Typical Appearance:* 	Not necessary for restoration of structural performance. (Measures may be necessary for restoration of nonstructural characteristics.)
Slight	Not used	

COMPONENT DAMAGE
CLASSIFICATION GUIDE **continued**

URM1F

Level	**Description of Damage**	**Typical Performance Restoration Measures**
Moderate $\lambda_K = 0.9$ $\lambda_Q = 0.6^1$ $\lambda_D = 0.9$ 1. As an alternative, calculate as V_{bjs2}/V_{tc} $\Delta/h_{eff} \leq 0.8\%$	*Criteria:* 1. Horizontal cracks/spalled mortar at bed joints at or near the base of the wall indicating that in-plane offset along the crack has occurred up to approximately 1/4". 2. Possibly diagonally-oriented cracks and spalling at the toe of the wall. Cracks extend upward several courses. 3. Possibly diagonally-oriented cracks at upper portions of the wall which may be in the units. *Typical Appearance:* 	• Replace/drypack damaged units. • Repoint spalled mortar and open head joints. • Inject cracks and open head joints. • Install pins and drilled dowels in toe regions. $\lambda_K{}^* = 1.0^1$ $\lambda_Q{}^* = 1.0^1$ $\lambda_D{}^* = 1.0^1$ 1. In some cases, grout injection may actually increase strength, but decrease deformation capacity, by changing behavior from bed joint sliding to a less ductile behavior mode (see FEMA 307, Section 4.1.3).
Heavy $\lambda_K = 0.8$ $\lambda_Q = 0.6^1$ $\lambda_D = 0.9$ 1. As an alternative, calculate as V_{bjs2}/V_{tc} $\Delta/h_{eff} \leq 1.2\%$	*Criteria:* 1. Horizontal bed joint cracks near the base of the wall similar to Moderate, except width is up to approximately 1/2". 2 Possibly extensive diagonally-oriented cracks and spalling at the toe of the wall. Cracks extend upward several courses. 3 Possibly diagonally-oriented cracks up to 1/2" at upper portions of the wall. *Typical Appearance:* 	• Replace/drypack damaged units. • Repoint spalled mortar and open head joints. • Inject cracks and open head joints. • Install pins and drilled dowels in toe regions. $\lambda_K{}^* = 1.0^1$ $\lambda_Q{}^* = 1.0^1$ $\lambda_D{}^* = 1.0^1$ 1. In some cases, grout injection may actually increase strength, but decrease deformation capacity, by changing behavior from bed joint sliding to a less ductile behavior mode (see FEMA 307, Section 4.1.3).
Extreme	*Criteria:* • Vertical load-carrying ability is threatened *Typical Indications* • Stair-stepped movement is so significant that upper bricks have slid off their supporting brick. • Toes have begun to disintegrate. • Residual set is so significant that portions of masonry at the edges of the pier have begun or are about to fall.	• Replacement or enhancement required.

URM2K	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**URM**
		Component Type:	**Weaker Pier**
		Behavior Mode:	**Diagonal Tension**

How to distinguish behavior mode:

By observation:

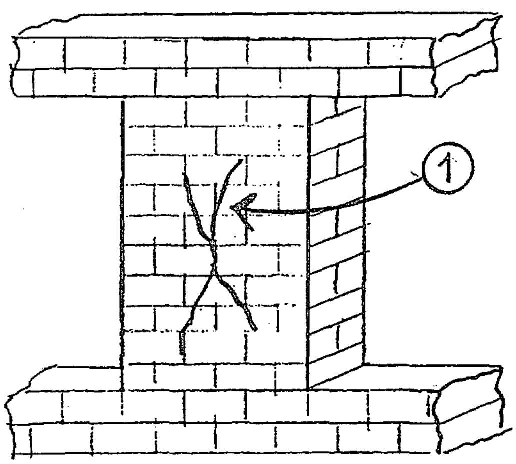
Typical diagonal tension cracking—resulting from strong mortar, weak units, and high compressive stress—can be identified by diagonal cracks ("X" cracks) that propagate through the units. In many cases, the cracking is sudden, brittle, and vertical load capacity drops quickly. The cracks may then extend to the toe and the triangles above and below the crack separate. In a few cases, the load drop may be more gradual with cracks increasing in size and extent with each cycle. A second form of diagonal tension cracking also has been experimentally observed with weak mortar, strong units and low compressive stress where the cracks propagate in a stair-stepped manner in head and bed joints. The first (typical) case is shown below.

By analysis:

Since the stair-stepping form of cracking would appear similar to the early levels of stair-stepped bed joint sliding, confirm by analysis that diagonal tension governs over bed joint sliding. Since deterioration at the corners in the Heavy damage level may resemble toe crushing, also confirm that diagonal tension governs over toe crushing.

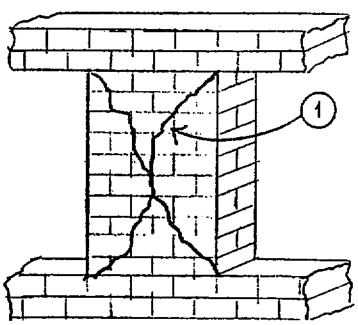
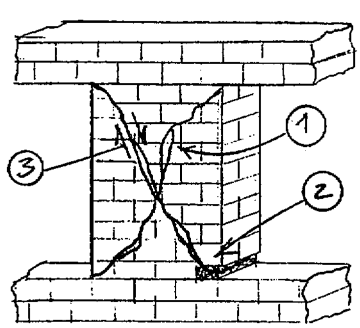
Refer to Evaluation Procedures for:

- In-plane wall behavior: See Section 7.3.2.

Level	Description of Damage	Typical Performance Restoration Measures
Insignificant $\lambda_K = 1.0$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$ $\mu_\Delta \leq 1$	*Criteria:* 1. Hairline diagonal cracks in masonry units in fewer than 5% of courses. *Typical Appearance:* 	Not necessary for restoration of structural performance. (Measures may be necessary for restoration of nonstructural characteristics.)

COMPONENT DAMAGE
CLASSIFICATION GUIDE **continued**

URM2K

Level	**Description of Damage**	**Typical Performance Restoration Measures**
Slight	Not used.	
Moderate $\lambda_K = 0.8$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$ $\mu_\Delta \approx 1\text{-}1.5$	*Criteria:* 1. Diagonal cracks in pier, many of which go through masonry units, with crack widths below 1/4". 2. Diagonal cracks reach or nearly reach corners. 3. No crushing/spalling of pier corners. *Typical Appearance:* 	• Repoint spalled mortar. • Inject cracks. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Heavy $\lambda_K = 0.4$ $\lambda_Q = 0.8$ $\lambda_D = 0.7$ $\mu_\Delta > 1.5$	*Criteria:* 1. Diagonal cracks in pier, many of which go through masonry units, with crack widths over 1/4". Damage may also include: 2. Some minor crushing/spalling of pier corners and/or 3. Minor movement along or across crack plane. *Typical Appearance:* 	Replacement or enhancement is required for full restoration of seismic performance. For partial restoration of performance: • Replace/drypack damaged units. • Repoint spalled mortar. • Inject cracks. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 0.8$ $\lambda_D{}^* = 1.0$
Extreme	*Criteria:* • Vertical load-carrying ability is threatened *Typical Indications* • Significant movement or rotation along crack plane. • Residual set is so significant that portions of masonry at the edges of the pier have begun or are about to fall.	• Replacement or enhancement required.

URM1H	DAMAGE CLASSIFICATION AND REPAIR GUIDE	System:	**URM**
		Component Type:	**Solid Wall**
		Behavior Mode:	**Flexural Cracking/ Toe Crushing**

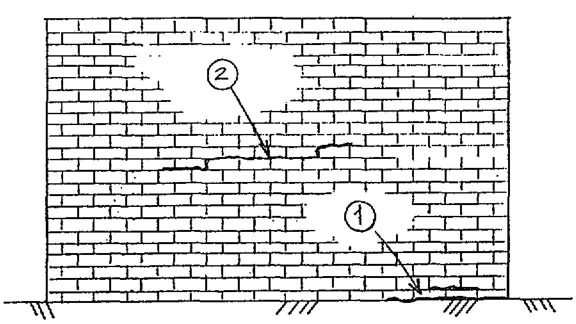
How to distinguish behavior mode:

By observation:
This type of behavior typically occurs in stockier walls with $L/h_{eff} > 1.25$. Based on laboratory testing, four steps can usually be identified. First, flexural cracking happens at the base of the wall, but it does not propagate all the way across the wall. This can also cause a series of horizontal cracks to form above the heel. Second, sliding occurs on bed joints in the central portion of the pier. Third, diagonal cracks form at the toe of the wall. Finally, large cracks form at the upper corners of the wall. Failure occurs when the triangular portion of wall above the crack rotates off the crack or the toe crushes so significantly that vertical load is compromised. Note that, for simplicity, the figures below only show a single crack, but under cyclic loading, multiple cracks stepping in each direction are possible.

By analysis:
Stair-stepped cracking may resemble a form of bed joint sliding; confirm by analysis that toe crushing governs over bed joint sliding.

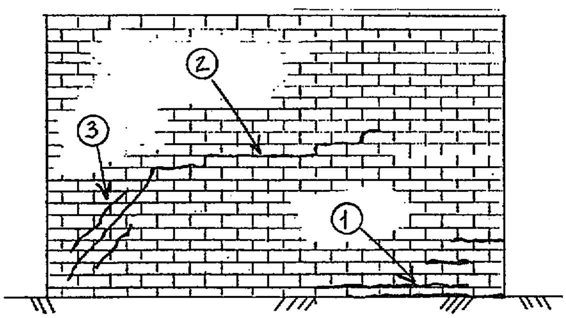
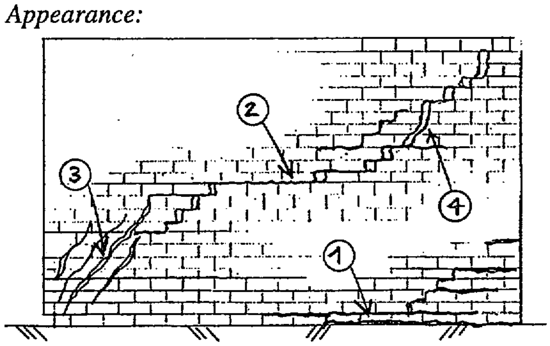
Refer to Evaluation Procedures for:

- In-plane wall behavior: See Section 7.3.2

Level	Description of Damage	Typical Performance Restoration Measures
Insignificant $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$ $\mu_\Delta \leq 1.5$	*Criteria:* 1. Horizontal hairline cracks in bed joints at the heel of the wall. 2. Horizontal cracking on 1-3 cracks in the central portion of the wall. No offset along the crack has occurred and the crack plane is not continuous across the pier. 3 No cracks in masonry units. *Typical Appearance:* 	Not necessary for restoration of structural performance. (Measures may be necessary for restoration of nonstructural characteristics.)
Slight	Not used	
Moderate	Not used	

COMPONENT DAMAGE
CLASSIFICATION GUIDE **continued**

URM1H

Level	**Description of Damage**	**Typical Performance Restoration Measures**
Heavy $\lambda_K = 0.8$ $\lambda_Q = 0.8$ $\lambda_D = 1.0$ $\Delta/h_{eff} \leq 0.3\%$	*Criteria:* 1. Horizontal hairline cracks in bed joints at the heel of the wall. 2. Horizontal cracking on 1-3 cracks in the central portion of the wall. Some offset along the crack may have occurred. 3. Diagonal cracking at the toe of the wall, likely to be through the units, and some of units may be spalled. *Typical Appearance:* 	Replacement or enhancement is required for full restoration of seismic performance. For partial restoration of performance: • Repoint spalled mortar. • Inject cracks. $\lambda_K{}^* = 0.9$ $\lambda_Q{}^* = 0.9$ $\lambda_D{}^* = 1.0$
Extreme $\lambda_K = 0.6$ $\lambda_Q = 0.6$ $\lambda_D = 0.9$ $\Delta/h_{eff} \leq 0.9\%$	*Criteria:* 1. Horizontal hairline cracks in bed joints at the heel of the wall. 2 Horizontal cracking on 1 or more cracks in the central portion of the wall. Offset along the crack will have occurred. 3 Diagonal cracking at the toe of the wall, likely to be through the units, and some of units may be spalled. 4 Large cracks have formed at upper portions of the wall. In walls with aspect ratios of $L/h_{eff} > 1.5$, these cracks will be diagonally oriented; for more slender piers, cracks will be more vertical and will go through units. *Typical Appearance:* 	Replacement or enhancement is required for full restoration of seismic performance. For partial restoration of performance: • Replace/drypack damaged units. • Repoint spalled mortar. • Inject cracks. • Install pins and drilled dowels in toe regions. $\lambda_K{}^* = 0.9$ $\lambda_Q{}^* = 0.8$ $\lambda_D{}^* = 1.0$

URM3I	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**URM**
		Component Type:	**Weak Spandrel**
		Behavior Mode:	**Spandrel Unit Cracking**

How to distinguish behavior mode:

By observation:

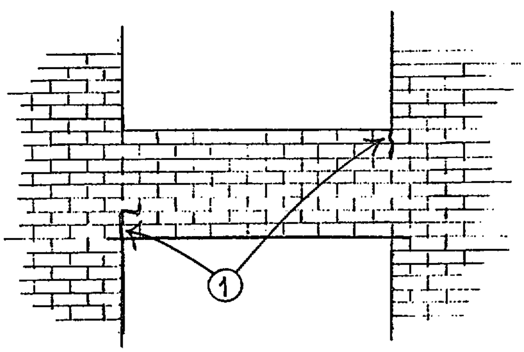
In this type of behavior, the moment at the end of the spandrel is not relieved by sliding, but instead causes brittle vertical cracking though the masonry units. Cracking propagates rapidly as displacement increases and cycles continue. Depending on the lintel construction, this can lead to a local falling hazard. It also increases the effective height of the piers. As the spandrel displaces, the nonlinear mechanism of response may move to other portions of the wall such as the piers.

By analysis:

No analysis is typically necessary to distinguish this mode.

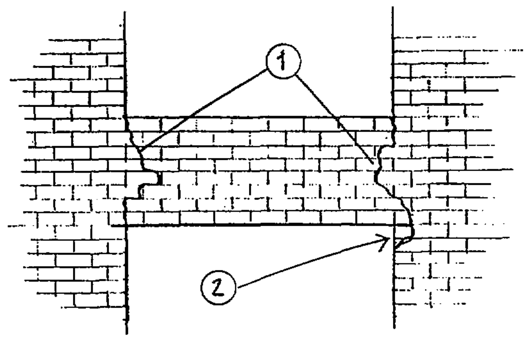
Refer to Evaluation Procedures for:

- In-plane wall behavior: See Section 7.3.4.

Level	Description of Damage	Typical Performance Restoration Measures
Insignificant $\lambda_K = 0.9$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$ $\mu_\Delta \leq 1.5$	*Criteria:* 1. Predominantly vertical cracks/spalled mortar through no more than one unit at the ends of the spandrel. *Typical Appearance:* 	Not necessary for restoration of structural performance. (Measures may be necessary for restoration of nonstructural characteristics.)
Slight	Not used.	

COMPONENT DAMAGE
CLASSIFICATION GUIDE **continued**

URM3I

Level	Description of Damage		Typical Performance Restoration Measures
Moderate	Not Used		
Heavy $\lambda_K = 0.2$ $\lambda_Q = 0.4^1$ $\lambda_D = 0.6$ 1. As an alternative, calculate per Section 7.3.4	*Criteria:*	1. Predominantly vertical cracks/spalled mortar across the full depth of each end of the spandrel. In over 1/3 of the courses, cracks go through the masonry units. 2. Possibly some deterioration of units at bottom ends of spandrel, but no vertical slip of the spandrel. *Typical Appearance:* 	Replacement or enhancement is required for full restoration of seismic performance. For partial restoration of performance: • Stitch across crack with pins and drilled dowels. • Repoint spalled mortar. • Inject cracks. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 0.8$ $\lambda_D{}^* = 1.0$
Extreme	*Criteria:* *Typical Indications*	• Vertical load-carrying ability is threatened. One or more of the following: • Lintel support has separated from the pier. • Out-of-plane movement of the spandrel. • Spandrel has slipped vertically.	• Replacement or enhancement required.

URM1M	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**URM**
		Component Type:	**Solid Wall**
		Behavior Mode:	**Out-of-Plane Flexural Response**

How to distinguish behavior mode:

By observation:

Out-of-plane failures are common in URM buildings. Usually they occur due to the lack of adequate wall ties, as discussed in Table 7-1. When ties are adequate, the wall may fail due to out-of-plane bending between floor levels. One mode of failure observed in experiments is rigid-body rocking motion occurring on three cracks: one at the top of the wall, one at the bottom, and one at midheight. As rocking increases, the mortar and masonry units at the crack locations can be degraded, and residual offsets can occur at the crack planes. The ultimate limit state is that the walls rock too far and overturn. Important variables are the vertical stress on the wall and the height-to-thickness ratio of the wall. Thus, walls at the top of buildings and slender walls are more likely to suffer damage.

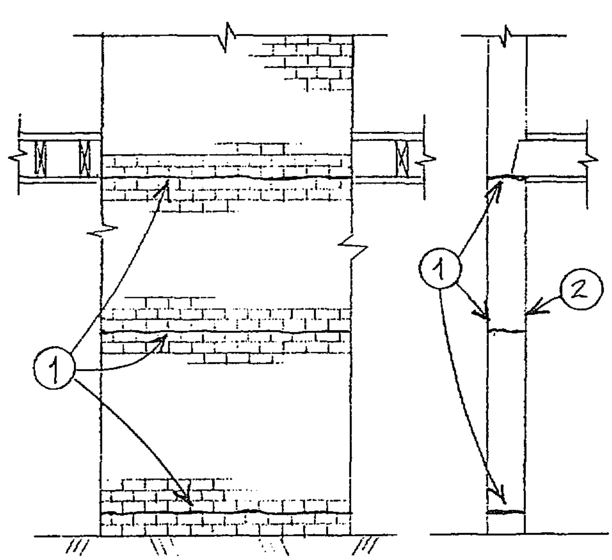
By analysis:

None required.

Caution:

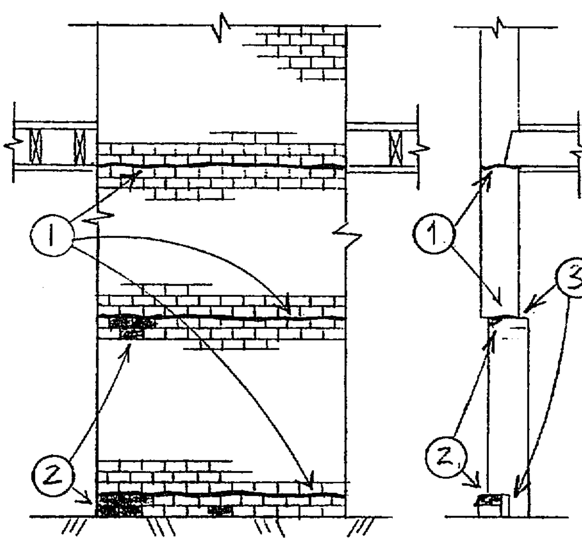
If horizontal cracks are located directly below wall-diaphragm ties, damage may be due to bed joint sliding associated with tie damage. For piers, if horizontal cracks are observed at the top and bottom of the pier but not at midheight, see URM2A. Confirm whether the face brick is unbonded to the backing brick. If so, the thickness in the *h/t* requirement is reduced to the thickness of the backing wythes.

Refer to Evaluation Procedures for: Out-of-plane wall behavior: See Section 7.3.5.

Level	Description of Damage	Typical Performance Restoration Measures
Insignificant For out-of-plane loads: $\lambda_{h/t} = 1.0$ For in-plane modes given previously, assume out-of-plane damage leads to Moderate damage for URM2B and Insignificant damage for all other modes.	*Criteria:* 1. Hairline cracks at floor/roof lines and midheight of stories. 2. No out-of-plane offset or spalling of mortar along cracks. *Typical Appearance:* 	Not necessary for restoration of structural performance. (Measures may be necessary for restoration of nonstructural characteristics.)

COMPONENT DAMAGE
CLASSIFICATION GUIDE **continued**

URM1M

Level	**Description of Damage**	**Typical Performance Restoration Measures**
Slight	Not used.	
Moderate For <u>out-of-plane loads</u>: $\lambda_{h/t}$= 0.9 For <u>in-plane modes</u>, see Insignificant damage	*Criteria:* 1. Cracks at floor/roof lines and midheight of stories may have mortar spalls up to full depth of joint and possibly: 2. Out-of-plane offsets along cracks of up to 1/8". *Typical Appearance:* See Insignificant damage above.	• Repoint spalled mortar: • For <u>out-of-plane loads</u>: $\lambda_{h/t}$= 1.0 • For <u>in-plane loads</u>: use Moderate for URM2B and Insignificant for all other modes.
Heavy For <u>out-of-plane loads</u>: $\lambda_{h/t}$= 0.6 For <u>in-plane modes</u> given previously, assume out-of-plane damage leads to *Heavy* for all other modes.	*Criteria:* 1 Cracks at floor/roof lines and midheight of stories may have mortar spalls up to full depth of joint. 2 Spalling and rounding at edges of units along crack plane. 3 Out-of-plane offsets along cracks of up to 1/2". *Typical Appearance:* 	Replacement or enhancement is required for full restoration of seismic performance. For <u>partial</u> restoration of out-of-plane performance: • Replace/drypack damaged units • Repoint spalled mortar $\lambda_{h/t}$= 0.8
Extreme	*Criteria:* • Vertical-load-carrying ability is threatened *Typical Indications* • Significant out-of-plane or in-plane movement at top and bottom of piers ("walking"). • Significant crushing/spalling of bricks at crack locations.	• Replacement or enhancement required.

8: Infilled Frames

8.1 Introduction and Background

This section provides material relating to infilled frame (INF) construction that supports and supplements the Damage Classification Guides (or Component Guides) in Chapters 5 through 7. Following this introductory material, infilled frame component types are defined and discussed in Section 8.2. Inelastic behavior modes are also summarized in Section 8.2. The overall damage evaluation procedure uses conventional material properties as a starting point. Section 8.3 provides information on strength and deformation properties of infilled frame components. The information on infilled frame components has been generated from a review of available empirical and theoretical data listed in the Tabular Bibliography (Section 5.2 of FEMA 307) and References section of this document, or Section 5.3 of FEMA 307). These provide the user with further detailed resources on infilled frame component behavior.

Infilled frame construction has been in use for more than 200 years. The infilling of frames, in contrast with URM structures, is associated primarily with the construction of high-rise buildings—the frames being a means of carrying gravity loads, the infills a means of providing a building envelope and/or internal partitioning. In high-rise structures, the frames have been generally well-engineered in accordance with the state-of-knowledge of the day, whereas the infill panels were invariably considered to be "nonstructural". It was not until the 1950s that investigations began on the interaction between infill panels and the frames of buildings (Polyakov, 1956). This pioneering work undertaken in the former Soviet Union was strictly a reflection of Russian building practice. However, many of the theoretical techniques and other findings are still of relevance. The first study in the United States that investigated the lateral-load behavior of infilled frames, using specimens typical of U.S. construction practice (steel frames with brick infills), was reported by Benjamin and Williams (1958). These and other early studies were mostly concerned with the monotonic lateral-strength capacity of infilled frame systems.

Immediately following the advent of experimental investigations, analytical research began on the performance of infilled-frame systems. Over the years, several different methods of analysis were proposed for determining the composite strength of an infilled-frame system. These methods included elasticity solutions based on the Airy stress function, the finite-difference method, the finite-element method, and plastic methods of analysis. For a summary, see Maghaddam and Dowling (1987).

Although these methods of analysis have been shown to be reasonably successful in predicting the strength capacity of infilled-frame systems, each method has its roots in elasticity or rigid plasticity, making it either difficult or impossible to extend the findings to inelastic (elasto-plastic) behavior, especially if cyclic loading is to be considered. Therefore, it is not surprising that the equivalent-strut method of analysis has become the most popular approach for analyzing infilled frame systems. Early equivalent-strut methods, starting with Stafford-Smith (1966), used an equivalent single strut to represent infill behavior. It was later realized that such a simplification did not accurately capture all facets of frame/panel interaction. Therefore, several multiple-strut methods of analysis have been proposed (see for example Chrysostomou et al., 1988; Thiruvengadam, 1985; Mander et al., 1994). In spite of these attempts to enhance infilled frame analysis using a multiple-strut approach, there are still drawbacks—principally the inability to model force transfer-slip at the frame-panel interfaces (Gergely et al., 1994). Nonlinear finite element analysis, however, can be used if such a refinement is required (Shing et al., 1994; Mosalan et al., 1994), but difficulties remain, mostly due to computational limitations, on analyzing more than one panel at a time.

The general consensus is that a single equivalent-strut approach (two struts per panel for reversed cyclic loading analysis, one across each diagonal) may be successfully used for design and evaluation studies of infilled frame systems. Such an approach has been recently adopted by FEMA 273.

In spite of the general success of modeling infilled frames with solid panels, major difficulties still remain unresolved regarding the modeling approach for infilled frames with openings. Such frames, in practice, are commonplace and are perhaps the norm rather than the exception. However, only a limited amount of research has been undertaken on infilled frames with openings (e.g., Benjamin and Williams, 1958; Durrani and Luo, 1994; Coul, 1966; Dawe et al., 1985a,b; Holmes, 1961; Liauw, 1977; Mallick and Garg, 1971). Other strength analysis recommendations have been made for infills with wide openings, but these have not been substantiated by experimental studies (Hamburger and Chakradeo, 1993; Freeman, 1994). For this reason, it is suggested that infilled frames with openings exceeding 50 percent of the panel area be treated either using other sections of this document (namely URM for infills with brick piers or reinforced concrete for cases in which infills surround steel columns) or by nonlinear finite element procedures such as discussed by Kariotis et al. (1996).

It is important to recognize that many behavior problems with infilled frames arise from discontinuities of infill, resulting from soft stories or checkered patterns, leading to a high concentration of forces to be transferred among components.

Other impediments to reliable modeling generalizations of infilled-frame systems are the large variation in construction practice over different geographic regions and changes of materials over time. Early infilled-frame construction generally consisted of clay brick (or sometimes stone masonry) and iron/steel frames. With time, concrete frames became popular and concrete masonry units or solid (poured) concrete were used for the infill panels. Concrete masonry or concrete infill panels may be either unreinforced or reinforced (and grouted or not in the case of concrete blocks).

Early research that investigated the seismic performance of infilled-frame specimens using reversed cyclic loading mostly focused on developing improved seismically-resistant design, analysis, and construction techniques for new structures (e.g., Axely and Bertero, 1979; Bertero and Brokken, 1983; Klingner and Bertero, 1976, 1978; Zarnic and Tomazevic, 1984, 1985a,b). Little research was done to investigate the seismic performance of existing structures with nonductile detailing. Although some studies have been conducted on infilled frames with deficient detailing (e.g., Gergely et al., 1993, 1994; Flannagan and Bennett, 1994; Mander et al., 1993a,b; Reinhorn et al., 1995), much work, especially experimental investigations, remains to be done.

8.2 Infilled Frame Masonry Component Types and Behavior Modes

8.2.1 Component Types

Infilled-frame elements are made up of infilled-panel and frame components, as summarized in Table 8-1.

The general characteristics of these basic components are summarized as follows:

a. Infilled Panels

Infilled panels are primarily categorized according to material and geometric configuration.

Materials. Clay brick masonry is perhaps the most commonly encountered type of infill material. The use of this traditional building material for infill construction dates back to the 1800s, when steel or iron frames were first used for high-rise construction. Generally twin or multi-wythe bricks are used, but other forms exist, such as cavity walls for exterior facades. Most often, brick masonry is unreinforced (see Section 7.1). In more modern buildings, reinforced, grouted-cavity wall construction may be found.

Hollow clay tile (HCT) is a relatively modern form of unreinforced masonry infill construction (see Section 7.1). The infills are often offset with respect to the centerlines of columns. HCT is often found on building facades. With steel frames, the clay tiles can be placed around the frames for aesthetic and fire protection purposes. HCT is very commonly used for interior partitions in framed buildings. As a material, HCT is generally very brittle and prone to force-controlled behavior.

Concrete masonry unit (CMU) construction is a form of infill using hollow concrete blocks laid up with mortar. CMU may be left hollow or filled with grout, either partially or completely. If grouted, steel reinforcement may or may not be present. The strength and ductility of the infill is highly dependent on the degree of grouting and reinforcement. Ungrouted infills are comparatively weak. This is because when the in-plane forces become large, compressive splitting of the face shells occurs with a complete loss of strength in masonry. Moreover, sliding-shear resistance relies entirely on the mortar in the bed joints. Grouted concrete masonry infills can be quite strong for normal bay sizes. Although early spalling of face shells may occur due to high in-plane lateral compression stresses, the grouted core has considerable ability in resisting additional loads, particularly if reinforced. Chapter 6 has additional information for reinforced CMU and Chapter 7 is applicable to unreinforced CMU.

Concrete infills are typically reinforced, though often minimally. In older buildings, the reinforcement is generally only for temperature and shrinkage control and is rarely provided to resist structural loads. In

Table 8-1 Component Types for Infilled Frames

Component Type		Description/Examples	Materials/Details
INPS	Solid infill panel	Space within frame components completely filled	Concrete Reinforced Unreinforced Masonry (clay brick, hollow clay tile, concrete block) Reinforced Unreinforced
INPO	Infill panel with openings	Doors and windows Horizontal or vertical gaps Partial-height infill Partial-width infill	Same as solid infill panel
		Sub-components similar to:	
	INP1 Strong pier	RC1 RM1 URM1	Concrete Reinforced masonry URM
	INP2 Weak pier	RC2 RM2 URM2	Concrete Reinforced masonry URM
	INP3 Weak spandrel (lintel)	RC3 RM3 URM3	Concrete Reinforced masonry URM
	INP4 Strong spandrel (lintel)	RC4 RM4 URM4	Concrete Reinforced masonry URM
INF1	Frame column	Vertical, gravity-load-carrying	Concrete Steel
INF2	Frame beam	Horizontal, gravity-load-carrying	Concrete Steel
INF3	Frame joint	Connection between column and beam components Rigid moment-resisting Partially-rigid Simple shear	Monolithic concrete Precast concrete Bolted steel Riveted steel Welded steel

modern buildings, however, the reinforced concrete infill may be well reinforced and act compositely with the surrounding frame. See Chapter 5 for additional information applicable to concrete infill.

Geometry. Infill may have a wide variety of geometric configurations. Aspect ratios (length/height of the planar space defined by the surrounding frame components) for infilled panels varies from approximately 1:1 to 3:1 with most ranging from 1.5:1 to 2.5:1. Infills may be configured in many forms to suit partitioning and/or facade requirements. It is not uncommon to find infills placed eccentrically to the axis of the frame components. For wider multi-wythe infills, entire rows of bricks may not engage with the frame at all. This leads to differential behavior and movements. For the purposes of evaluating infilled frame performance, only those bricks/blocks bounded by frames should be considered as part of the load infilled panel component.

For the purposes of damage evaluation, Table 8-1 identifies two categories for infilled panel components based on geometric configuration. Solid infilled panel components (INPS) are those that completely fill the planar space tightly within the surrounding frame components. Those with openings (INPO) may exhibit fundamentally different behavior.

Initial gaps at the top or sides of an infill affect performance of the solid panel configurations. These gaps can arise from the construction process not providing a tight infill, or in the case of concrete masonry units, from shrinkage. Until gaps are closed, normal frame behavior can be expected. When the gaps suddenly close, impacting forces on the infill can dramatically change the behavior patterns of the frame. Seismic gaps can be built into infill wall panels, although this practice is not common in the United States.

Perforations within the infill panels are the most significant parameter affecting seismic behavior of infilled systems. Doors and windows are the two most prevalent opening types. Openings located in the center portion of the infill can lead to weak infill behavior. On the other hand, partial-height infills (with windows spanning the entire top half of the bay) can be relatively strong. The frames are often relatively weak in column shear and when partial-height infill is present, this potentially leads to a short-column soft-story collapse mechanism. Partial-width infills are also relatively common: in this case, window openings extend the full height between floors. Partial-width infill often has been placed on each side of a column component.

b. Frames

The frame components of infilled-frame seismic elements are categorized primarily by material.

Steel. Steel frames are common, especially for older structures. Steel frames are also popular for modern high-rise buildings and low-rise, light-weight, commercial, building construction. Column and beam components are most often I-sections (older) or wide-flange sections (newer). Built-up columns and double-channel beams are much less common. In older steel frames, the beam and column components are typically joined by semi-rigid riveted connections. More modern steel frames often use bolted or welded, or both, semi-rigid connections. Many of the frame systems are enclosed by concrete, the beams enclosed as a part of the floor system and the columns encased for fire protection. In these circumstances semi-rigid (or partially restrained) riveted connections will behave as fully restrained until the confining concrete cracks. Because of the relatively high shear capacity of steel columns, the fully restrained mode of behavior may be dominant.

Concrete. Concrete frames are also a common form of construction. Reinforced concrete frames may be classified as either ductile or nonductile for seismic performance, based primarily on the details of reinforcement. Contemporary structural design requires ductile detailing of the members. Ductile detailing requires closely-spaced transverse hoops in the beams, columns, and connections. If such members surround weak infill panels, they will suffer relatively less serious damage under lateral loads. Fully-ductile frames are relatively rare and, in the United States, are found only in the west, and seldom in infilled frame buildings. Typically, beams do not have adequate confinement and rarely is the bottom reinforcement continued through the joint. Concrete columns can, however, be designed with ductility, particularly in the western United States, where it is not uncommon for the better-built buildings to have spiral reinforcing. Under these circumstances the columns possess a relatively high shear capacity and displacement ductility, and the beam or the infill will be the deformation-limiting component.

Non-ductile frames are very common, particularly in regions of low-to-medium seismic risk. These frames are not detailed for ductility and may have one or more deficiencies: columns weaker than beams, lap splices in column hinge zones, and insufficient transverse reinforcement for confinement, for shear strength, and for longitudinal reinforcement stability. The beam/column joints in concrete frames need to transmit high shear forces. When infills are present, shear force demands are considerably higher, leaving the beam or column vulnerable to shear failure.

Precast, prestressed, concrete frames are also commonly encountered with infilled panels. Although in many respects similar to reinforced concrete, the connections between columns and beams in precast construction are distinctly different. When non-engineered infills are placed between columns, premature failure may occur at the beam/column connections, leading to unseating of the beams.

8.2.2 Panel and Frame Modeling and Interaction

Because of the highly nonlinear nature of the infill/frame interaction, proper modeling of the behavioral characteristics is best accomplished by a thorough analysis, material testing, and nonlinear, finite-element, modeling. Lacking the resources for that approach, an estimate of the behavior may be made by using a procedure similar to the identification of the appropriate inelastic lateral mechanism, as discussed in Section 2.4 of this volume. The frame is modeled conventionally as an assembly of column (indicated by INF1) and beam (INF2) components, and connection components (INF3). The solid infilled components (INPS) can be modeled as equivalent struts in accordance with the recommendations of FEMA 273. Infilled components with openings (INPO) can sometimes also be modeled as struts depending on the size and location of the openings. Alternatively, sub-component "piers" (INP1, INP2) and "spandrels" (INP3, INP4) can be used to represent the infilled component with openings. Appropriate force-deformation characteristics for the sub-components can be generated using the information in Chapter 5 for concrete, Chapter 6 for reinforced masonry, and Chapter 7 for unreinforced masonry.

To establish the inelastic force-deformation behavior of the frame and infill using the component method, the engineer must manually determine the bifurcation points defining the mode of behavior. Broadly speaking, the behavior can be separated into two conditions which depend primarily on the degree of the infill interaction with the frame. In the case where the openings are extensive, the components can be assembled as frame elements and piers, with the frame performance modified by a potential for beam shear failure for cases with the infill primarily around the column, or by short-column effects where the infill is primarily around the beam. Small piers within the frame contribute little to the overall stiffness, but must be checked to ensure displacement compatibility with the frame-limited deformations.

For conditions where the infill is the controlling element, the degree of interaction is more complex. Initially, the defining characteristic is an uncracked panel. As the loading increases, the panel will experience bed-joint sliding or diagonal tension failure and transform the infill into an equivalent strut. Beam and column shears need to be investigated at this loading to ensure they are not the load-limiting condition. Following strut formation, corner crushing is often the next and final limiting condition. When checking corner crushing, the beam and column need to be checked for shear, and the column needs to be checked to verify that it has sufficient tension capacity to support the corner crushing. The tensile capacity is usually adequate for steel columns, but may be the limiting factor for lightly reinforced concrete columns, or columns having lap-splice problems.

8.2.3 Behavior Modes

a. Solid Panels

In cases where the infill component controls the stiffness, the events that define the shape of the force-deformation curve are bed-joint sliding, diagonal tension, corner crushing, general shear failure, and out-of-plane failure. Under small deformations the stiffness and behavior are dominated by the panel stiffness characteristics. As the deformation increases the panel characteristics will be a function of its element properties. When the masonry units are strong relative to the mortar, diagonal tension will result in a stair-stepped pattern of cracks through head and bed joints. When the mortar is stronger than the units (rare), cracks will develop through the units as well as the mortar and follow a line normal to the direction of the principal stress. With the stair-stepped cracks, shear can continue to be resisted after cracking by the development of a compressive stress normal to the bed joints, characterized as a compression strut. If the mortar is weak relative to the units, an infill panel may crack along the bed joints instead of along the diagonal. In this case, horizontal cracks may occur across several bed joints as an assembly of units slides to accommodate the deflected shape of the frame. Although this cracking mode may occur at lower shear forces, the overall frame-infill will possess greater inelastic deformation capacity because frame action will dominate. When the infill panel is sufficiently strong in shear, the compressive stress at the corners will fail in crushing. This mode will be the strongest and stiffest, but has limited deformation capacity because the crushing will be abrupt. Furthermore, the large forces generated in this mode will be distributed to the beam and column members, and may result in either column or beam shear failures. Table 8-2 presents four principal behavior modes for solid-infill panel components. Further explanation on the expected damage characteristics and likelihood of occurrence are given below.

i. Bed-Joint Sliding: This behavior mode commonly occurs in conjunction with other modes of failure. Bed-joint sliding is likely to occur when the bounding frame is strong and flexible (such as steel frames). If the mortar beds are relatively weak compared to the adjacent masonry units (especially bricks), a plane of weakness forms, usually near the mid-height level of the infill panel. Damage takes the form of minor crushing. There is really no limit to the displacement capacity of this behavior mode. Therefore, energy is continuously dissipated via Coulomb friction.

ii. Diagonal Cracking: Under lateral in-plane loading of an infill frame system, high compression stresses form across the diagonal of an infill. Transverse to these principal compression stresses and strains are tension strains. When the tensile strains exceed the cracking strain of the infill panel material, diagonal cracking occurs. These cracks commence in the center of the infill and run parallel to the compression diagonal. As interstory drifts increase, the diagonal cracks tend to propagate until they extend from one corner to the diagonally opposite corner. This common form of cracking is evident in most infill panels that have been subjected to high lateral loads and sometimes occur with bed-joint sliding. Diagonal cracking behavior usually signals the formation of a new diagonal strut behavior mode.

iii. Corner Compression: Under lateral loading of infilled frames, some form of corner compression inevitably occurs. This is because of the high stress concentrations at each corner of the compression diagonal. For strong/stiff columns and beams, corner crushing is located over a relatively small region; whereas for weaker frames, especially concrete frames, corner crushing is more extensive and the damage extends into the concrete frame itself. In spite of the crushing damage that occurs, this is a relatively ductile failure mode. As interstory drifts increase, corner crushing becomes more pronounced to the extent that masonry units in the corner may fall out entirely. When this happens, crushing propagates towards the center of the beam and/or column.

iv. Out-of-Plane Failure: Ground shaking transverse to the plane of a wall may lead to an out-of-plane behavior mode. Experiments using air bags (Abrams, 1994), as well as shaking-table studies (Mander et al., 1994), show that for normal, infill panel, height-to-thickness ratios, considerable shaking is necessary to cause failure of the infill. However, out-of-plane failure may occur in the upper stories of high-rise buildings, where the floor accelerations are basically resonance amplifications of prominent sinusoidal ground motion input. In lower stories, when combined with high in-plane story shears, infill panels tend to progressively "walk-out" of the frame enclosure on each cycle of loading. Although complete out-of-plane failure is not common, there is some evidence that this behavior mode has occurred.

Table 8-2 Behavior Modes For Solid Infilled Panel Components

Behavior Mode	Description/Likelihood of Occurrence	Ductility	Figure	Paragraph (Section 8.2.3a)
Bed-joint sliding	Occurs in brick masonry, particularly when length of panel is large relative to height aspect ratio is large and the mortar strength is low.	High	8-2	i
Diagonal cracking	Likely to occur in some form	Moderate	8-1, 8-4, 8-5	ii
Corner compression	Crushing generally occurs with stiff columns.	Moderate	8-1	iii
Out-of-plane failure	More likely to occur in upper stories of buildings. However, out-of-plane "walking" is likely to occur in the bottom stories, due to concurrent in-plane loading.	Low	8-5	iv